Chapter 2
Design Considerations for Beach Stabilization Structures

2-1. General Design Objectives

a. Structural versus nonstructural alternatives.

(1) Beach stabilization structures alone do not provide the sand to maintain a wide protective or recreational beach; they simply redistribute available sand. Thus, accretion in one area is balanced by erosion elsewhere unless additional sand is introduced into the project area. The design of shore protection without concomitant beach nourishment must recognize that more sand in one area often means less in another area. The degree of allowable adverse effects needs to be addressed; however, if negative impacts cannot be tolerated, beach nourishment must be included in the project.

(2) Beach and dune restorations are often vulnerable and short lived due to the frequency and intensity of coastal storms. In addition to providing protection, however, they also contribute additional sand to the littoral environment. Frequent renourishment may be necessary to maintain a given level of protection. Coastal structures placed in conjunction with beach nourishment can often increase the residence time of the sand, keeping it on the beach within the project area for a longer period of time. If the savings realized by reducing the time between required renourishment exceeds the cost of the structures, their construction can be justified.

b. Alternative types of beach stabilization structures.

(1) Shore-parallel, onshore structures. Several types of beach stabilization structures can be built parallel to shore on an existing or restored shoreline. Revetments, bulkheads, and seawalls protect the area immediately behind them, but afford no protection to adjacent areas nor to the beach in front of them. While revetments, bulkheads, and seawalls can modify coastal processes such as longshore transport rates, cross-shore distribution of longshore transport, and onshore-offshore transport on the beach in front of them (if they protrude into the zone of longshore transport), these modifications do not affect their intended function, which is to protect the property behind them. These structures stabilize a shoreline by enclosing and protecting an area, thereby preventing the beach from functioning normally. The function and design of revetments, bulkheads, and seawalls is discussed in EM 1110-2-1614.

(2) Shore-connected structures.

(a) Groins and shore-connected breakwaters comprise the two types of beach stabilization structures in this category. Groins are the most common shore-connected beach stabilization structures. They are usually built perpendicular to shore to interrupt the normal transport of sand alongshore. Wave-induced longshore currents move sediment and cause it to accumulate in a fillet along the groin's updrift side (the side from which the sediment is coming). The groin also shelters a short reach of shoreline along its downdrift side from wave action. The accumulation of sand in a fillet along the updrift side of the groin reorients the shoreline and reduces the angle between the shoreline and the prevailing incident waves. This reduces the local rate of longshore sand transport and results in accumulation and/or redistribution of sand updrift of the groin and a reduction in the amount of sand moving past the groin. Diminished sand transport past a groin reduces the amount of sand contributed to the downdrift area and often causes erosion. Frequently, several groins are spaced along a beach to stabilize a long reach of shoreline. The groin system may or may not include a beach fill. If not artificially filled, natural longshore transport processes must fill the system. During the time the groins are filling, sand transport to downdrift beaches will be significantly reduced. This interruption of the natural sediment supply will cause erosion at the downdrift beaches. Unless special conditions warrant, prefilling the groin system should be considered mandatory.

(b) While groins are most often shore-perpendicular, they may sometimes be hooked or curved, or they may have a shore-parallel T-head at their seaward end. Hooked or curved groins are built in an attempt to increase the size of the updrift fillet or to shelter a greater stretch of beach from storm waves approaching from a predominant direction. A T-head groin may function primarily as a groin or as an offshore breakwater depending on the length of the T-head, structural transmissibility, and distance from shore. The T-head is often built to interrupt the seaward flow of water and sand in rip currents that often develop along a groin's axis. The T-head may also act as a breakwater and shelter a sizeable stretch of beach behind it.

(c) Important parameters that must be determined in designing a groin or groin system include: length, height and profile, planform geometry, spacing alongshore, type and materials of construction, permeability to sand, and the proposed fill sand's gradation.

(d) Shore-connected breakwaters extend seaward from shore and protect a stretch of beach from wave action. The quiet water behind the breakwater precludes erosion and, if sediment is in transport, allows it to accumulate in the structure's lee. Shore-connected breakwaters are generally dog-leg shaped in plan with a shore-connecting leg and a nearly shore-parallel leg; the shore-connecting leg often functions like a groin. They are often of either rubble-mound or sheet-pile construction. Frequently, shore-connected breakwaters are built to provide shelter for a marina rather than to provide shore stabilization. Shore stabilization and sedimentation effects are secondary, and the resulting sedimentation is often unwanted.

(3) Nearshore, shore-parallel breakwaters.

(a) Shore-parallel, detached (not shore-connected) breakwaters may be built singly or in series spaced along the shoreline. Detached breakwaters are constructed close to shore to protect a stretch of shoreline from low to moderate wave action and to reduce severe wave action and beach erosion. Sand transported along the beach is carried into the sheltered area behind the breakwater where it is deposited in the lower wave energy region. Protection afforded by the breakwater will limit erosion of the salient during significant storms and promote growth during periods of low to moderate wave activity. The effectiveness of a nearshore breakwater or breakwater system depends on the level of wave protection and the length of the shoreline it protects; thus, the breakwater's height, length, wave transmission characteristics, and distance from shore contribute to its effectiveness. For a system of breakwaters, the width of the gap between adjacent breakwaters and the length of the individual breakwater segments are also important.

(b) Nearshore breakwaters can also be constructed to create artificial headlands and are referred to as artificial headland breakwaters. In nature, where headlands are closely spaced and a limited sediment supply exists, small pocket beaches are formed (Chew et al. 1974). Pocket beaches are in hydraulic equilibrium, inherently stable, and recover rapidly after storm events (Hardaway and Gunn 1991). Where natural headlands are far apart and an adequate sediment supply exists, long and wide beaches are formed. Most headland beaches are between these extremes and assume a shape related to the predominant wave approach: a curved upcoast section representing a logarithmic spiral and a long and straight downcoast section (Chew et al. 1974). Headland beaches are often termed log-spiral beaches, crenulate-shaped, or pocket beaches. As opposed to detached breakwaters where tombolo formation is often discouraged, an artificial headland breakwater is designed to form a tombolo. Artificial headland design parameters include the approach direction of dominant wave energy, length of individual headlands, spacing and location, crest elevation and width of the headlands, and artificial nourishment.

(4) Shore-parallel offshore sills (perched beaches).

(a) Submerged or semisubmerged, shore-parallel offshore sills have been suggested as shore protection structures that can reduce the rate of offshore sand movement from a stretch of beach. The sill introduces a discontinuity into the beach profile so that the beach behind it is at a higher elevation (and thus wider) than adjacent beaches. The beach is thus "perched" above the surrounding beaches. This sill acts as a barrier to reduce offshore sand movement and causes some incoming waves to break at the sill. The sill functions like a nearshore breakwater by providing some wave protection to the beach behind it, although this sheltering effect is generally small since the sill's crest is relatively low. The height of the sill's crest and its alongshore continuity differentiates submerged sills from nearshore breakwaters. The crest of the submerged sill is usually continuous and well below normal high-tide levels; in fact, it is usually below low-tide levels.

(b) The low sill/perched beach concept minimizes the visibility of the structure since the sill crest is below the water's surface most of the time. Even when visible at low tide, it often remains more aesthetically acceptable than a detached breakwater. A disadvantage of the sill, however, is its potential as a hazard to swimming and navigation.

(5) Other. In many coastal locations, shore stabilization structures are already in place, having been built in response to a continuing erosion problem. These structures often have been modified over the course of their lifetime in attempts to improve their performance or to mitigate any adverse effects they might have caused. These modifications often account for the strange configurations of many structural shore stabilization systems found along eroding shorelines. For example, groins may initially have been built and subsequently modified by the addition of spurs (a diagonal extension off the structure),

hooked sections, or T-heads to reduce offshore sediment losses. Multiple-groin systems may have been extended downdrift along the coast in response to the progressive downdrift displacement of an erosion problem due to reducing the natural sand supply by updrift groin construction.

c. Selection among alternatives. Three major considerations for selecting among alternative beach stabilization schemes are: the primary and secondary objectives of the project, the physical processes prevailing at the project site, and the potential for adverse impacts along adjacent beaches. Appendix B provides descriptions of some of the advantages and disadvantages for various beach stabilization schemes.

(1) Primary and secondary objectives. Several factors determine what measures best meet the objectives of a given project. An important first step in selecting among alternative stabilization schemes is to carefully define the project's primary objective and any secondary objectives.

(a) A project's primary objective may be to protect inland development, maintain a beach, or both. Structures that armor the shoreline, beach stabilization structures, beach nourishment, or a combination of these may satisfy a project's primary objective. If the objective is simply to protect inland development from storm damage and to armor the shoreline against further erosion, a purely hard structural solution using a revetment or seawall might suffice. A beach seaward of the protective structure may or may not be important. If the objective is to protect inland development while maintaining a beach for additional protection and/or recreation, a solution involving either shore protection structures fronted by a beach fill, beach fill alone, or beach fill with stabilization structures might be sought. If the primary objective is to provide a protective beach or to stabilize an existing beach, then beach fill alone or beach fill with stabilization structures may be the solution.

(b) Secondary project objectives should also be identified and can often lead to additional project benefits. For example, a project's primary objective may be protection; however, a wide protective beach may also provide recreational benefits. Similarly, a project's primary objective may be to maintain a recreational beach, which will also afford some protection to back-beach development.

(2) Physical processes. Selecting an alternative shore protection/beach stabilization scheme also depends on the physical processes that prevail at a project site. If beach stabilization is a project's primary objective and net sediment losses from the project area are mainly by longshore transport, groins may provide a solution. On the other hand, if sediment losses are primarily offshore, groins cannot slow offshore losses; but, may exacerbate offshore diversion of sand by inducing rip current formation. Nearshore breakwaters reduce both alongshore and offshore sand losses, but significantly reduce wave conditions along the beach. Lower surf may or may not be desirable depending on intended beach use.

(3) Adverse impacts along adjacent beaches. The effect of a project on adjacent beaches is also a factor in selecting from among various types of shore stabilization. Structures such as groins and nearshore breakwaters, which reduce or for a time totally halt longshore transport, can cause erosion both downdrift and updrift of a project area. This impact can be avoided or mitigated by including beach nourishment as a part of the project. Including beach fill reduces the time it takes for the project to establish a new equilibrium beach planform configuration. It can take several years for a new equilibrium to be established if sand must be supplied by natural longshore sand transport alone. Beach fill thus encourages earlier sand bypassing of the project and reduces downdrift erosion. Where possible, groins and nearshore breakwaters should be designed to allow some sand bypassing to help alleviate downdrift erosion. If downdrift erosion is of no concern (such as the downdrift end of an island or a beach adjacent to a rocky shore), groin compartments and the beach behind nearshore breakwaters can be allowed to fill by natural longshore transport, if sufficient sediment is naturally available.

(a) Groins. Groins control the rate of longshore sand transport through a project area and reduce the rate of sand lost alongshore to downdrift beaches. If properly designed, they are effective in stabilizing beaches where sand is lost by alongshore movement. Groins function regardless of the direction of longshore transport and may exhibit seasonal variations in the location of the sand fillet as it shifts from one side of the structure to the other depending on the prevailing wave direction. Their effects often occur some distance both updrift and downdrift of the structure. Thus a single, relatively small groin can accumulate sand along a relatively long stretch of shoreline; likewise, erosion effects can often occur some distance downdrift of the structure. Groins are relatively easy to construct using land-based construction equipment and are also relatively easy to inspect and maintain. Groins do not significantly alter the characteristics of the waves along the beach except for a relatively limited area near the groin itself. They may cause offshore losses of sand during periods of high waves and water levels by deflecting longshore currents seaward. Wave setup in the compartment between two groins is greater on the updrift side of the downdrift groin, since waves there are larger and the shoreline is not sheltered by the structure. This condition causes a circulation within the compartment and may cause a rip current along the groin that can carry sand seaward. If sand losses from a beach are by offshore movement, groins will be ineffective in controlling erosion. Like all structures, groins alone do not provide sand; they simply redistribute available sand. Thus, sand held in an updrift fillet is kept from downdrift beaches, resulting in increased downdrift erosion rates. This problem can be avoided or delayed by including beach fill and nourishment as part of a groin project.

(b) Nearshore breakwaters. Nearshore breakwaters are effective shoreline stabilization structures that control both alongshore and offshore movement of sediment. They can be designed either singly or as a system of segmented breakwaters depending on the length of shoreline to be protected. There has been limited US experience with nearshore breakwater design, construction, and performance; thus, there is limited documented experience on which to base a design. The amount of longshore transport moving along a beach can be controlled by adjusting the length and spacing of the breakwater segments; however, unless the segments are carefully designed, nearshore breakwaters can disrupt longshore transport and starve downdrift beaches. Also, if built too close to shore, a tombolo (a sand spit extending from shore out to the offshore breakwater) can develop. The tombolo and breakwater can act as a groin, creating a total block to longshore sand transport until a new equilibrium is reached and bypassing resumes. Nearshore breakwaters significantly change the nature of the surf zone and the characteristics of the waves along a beach. Large waves break seaward of the breakwaters and only low, diffracted waves reach the beach behind the breakwaters. Waves acting on the structure may cause toe scour on the seaward side, and since the structures are located in shallow water nearshore, they are often subjected to the full force of breaking waves. Design wave conditions may be more severe than for revetments and seawalls onshore. Nearshore breakwaters are relatively expensive to construct because of their offshore location. Construction can be from the water using barges, from a temporary trestle, or from a temporary embankment built out from shore to the breakwater site. This embankment may later become part of a beach fill associated with the project. Likewise, inspection, maintenance, and repair will be more difficult and expensive than for land-connected structures.

(c) Beach fill. Beach fill and periodic nourishment are the only solutions to beach erosion problems that actually provide additional sand for a beach. Fill sand is usually obtained from a location some distance from the nourished beach: either an inlet, backbay area, or, in recent years, offshore or imported sources. It is often coupled with other shore protection measures to provide additional protection and recreation. Beach fills are often designed to provide a protective beach--a barrier of sand between the ocean and any back-beach development. Unless measures are taken to retain the beach fill and increase its residence time within a project area, beach fills may be short-lived. The presence of the fill does not appreciably alter the wave and nearshore current environment, and thus the erosion-causing factors continue unabated. Periodic nourishment is necessary to maintain a given level of protection. Depending on the size distribution of the fill sand relative to the native sand, erosion of the beach fill may be faster or slower than the original prefill erosion rate. Beach-stabilizing structures are built in conjunction with beach-fill projects to increase the residence time of the sand within the project area. As nearby sources of good quality beach sand are depleted, the cost of beach nourishment will increase since more distant sources must be exploited. Because of increasing costs of fill, stabilizing structures are becoming more economical. Structures are justified if they decrease the frequency of required periodic nourishment (increase the residence time between fills) so that nourishment is required less often. The anticipated savings accrued by less frequent nourishment should exceed the cost of structures.

d. Types of construction. Beach stabilization structures may be built of various materials and in various configurations. Factors such as the functional performance, cost, durability, and expected functional lifetime of an installation determine what type of construction is best.

(1) Rubble-mound construction. Groins, breakwaters, and offshore sills (perched beaches) are commonly constructed of quarrystone. Generally, rubble-mound structures comprise the most common type of coastal construction because they are able to dissipate most incident wave energy, thus reducing wave transmission and reflection. They are also "flexible" structures that do not lose their ability to function even when occasionally subjected to waves larger than the conditions for which they were designed. Failure is usually slow and progressive rather than catastrophic as it might be for more rigid structures.

(a) The design of rubble structures is described in EM 1110-2-2904 and the *Shore Protection Manual* (SPM 1984). Basically, the structure's outer or armor layer is built of quarrystone large enough to withstand selected design wave conditions at a selected design water level. The first underlayer (the layer of stone beneath the armor layer) is sized large enough so that it will not fit through the voids between the elements of the overlying layer. Each successive underlying layer is just large enough to be retained under the layer above it until quarry-run stone can be used in the cores. Armor stone is carefully placed and keyed to achieve maximum stability; however, it should be placed with sufficient voids so that incident wave energy is dissipated by turbulence within the structure's interstices. Figure 2-1 shows a typical quarrystone rubble-mound structure.

(b) When designing structures for the coastal environment, there is always some probability that design conditions will be exceeded during the structure's lifetime. Rubble structures may experience damage under such conditions and still maintain their ability to function. Rubble structures are often designed for the 10-percent wave height or the significant wave height (the average height of the highest 10 percent of the waves or the average height of the highest 33 percent of the waves, respectively) occurring during a storm with a given return period. At any instant in time during that storm, a range or distribution of wave heights prevails with occasional waves that exceed the 10-percent or significant height (about 18 percent of the waves in the distribution exceed the significant height). Consequently, rubble structures need not be designed to withstand the highest wave in the spectrum for the storm with a given return period.

(c) Information on potential sources of construction materials such as concrete aggregates and armor stone for rubble structures along with information on the quality of those materials is needed to select from among various structural alternatives. The location of the source relative to the construction site determines the cost of transportation. The weathering ability and durability of armor and underlayer stone and the chemical composition of concrete aggregates can have significant impact on the structural performance and service lifetime of a coastal structure. Information on the yield of potential quarries, the maximum size, and the size distribution of armor stone and underlayer stone a quarry will produce should be used to design rubble-mound structures that maximize the use of the quarry's production in the structure's cross section. A disadvantage of rubble structures is their relatively high construction costs and possibly the limited availability of suitable stone near many project sites. Also, if a distant quarry must be used, stone transportation costs may be high.

(d) When quarrystone heavy enough for the required armor is not available or when weight limits preclude transporting armor stone over public highways, precast concrete armor units may be an acceptable alternative. A wide variety of concrete armor unit shapes have been developed (SPM 1984, EM 1110-2-2904). Concrete armor units generally have improved stability characteristics that lead to comparable levels of stability with lighter, smaller units.

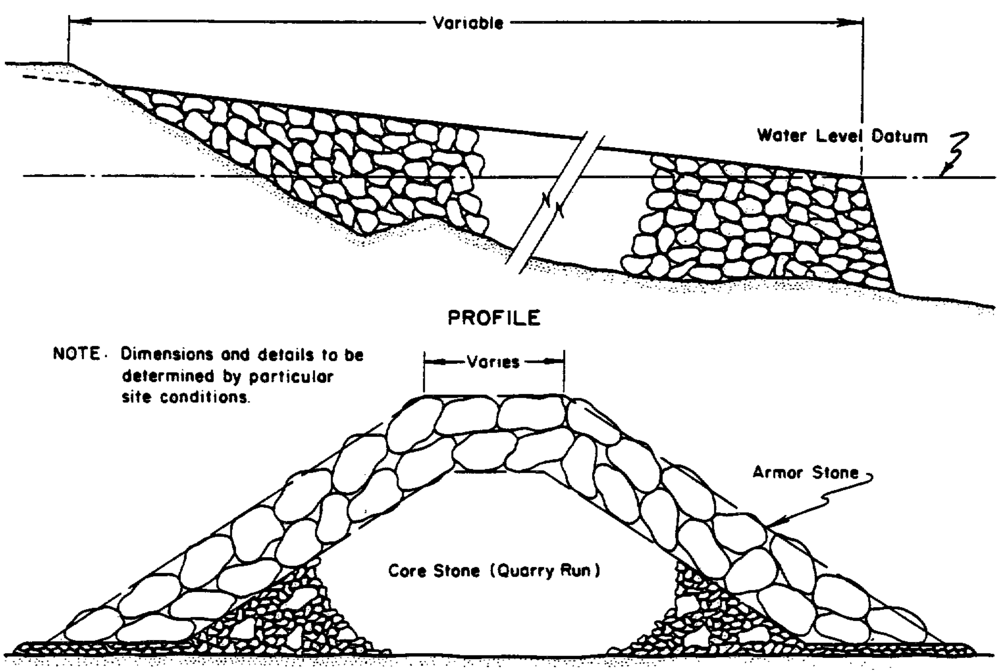
(2) Sheet-pile construction.

(a) Many functional groins, jetties, bulkheads, and in some cases, breakwaters and offshore sills have been built of sheet piling. Commonly, sheet piling used for shore protection has been timber, concrete, or steel. Sheet-pile structures usually have a relatively low initial cost since the volume of materials required is small, materials are readily available, and construction is usually faster than for comparable rubble structures. However, the service lifetime of these structures is often shorter, and therefore the life cycle cost may actually be higher. Sheet-pile structures are more rigid than rubble-mound structures and sustain damage if subjected to waves that exceed their design conditions. With the possible exception of good-quality concrete, the materials of which sheet pilings are made are less durable than stone in the marine environment. Deterioration and damage to sheet-pile structures often leads to a significant reduction in their ability to function properly.

(b) Sheet-pile structures reflect incident waves unless measures are taken to reduce their reflectivity. Often reflectivity is reduced by providing rubble along the structure. This rubble toe also serves as a scour blanket to prevent bottom scour. If wave reflections will not interfere with a structure's performance, sheet-pile structures may have an economic advantage.

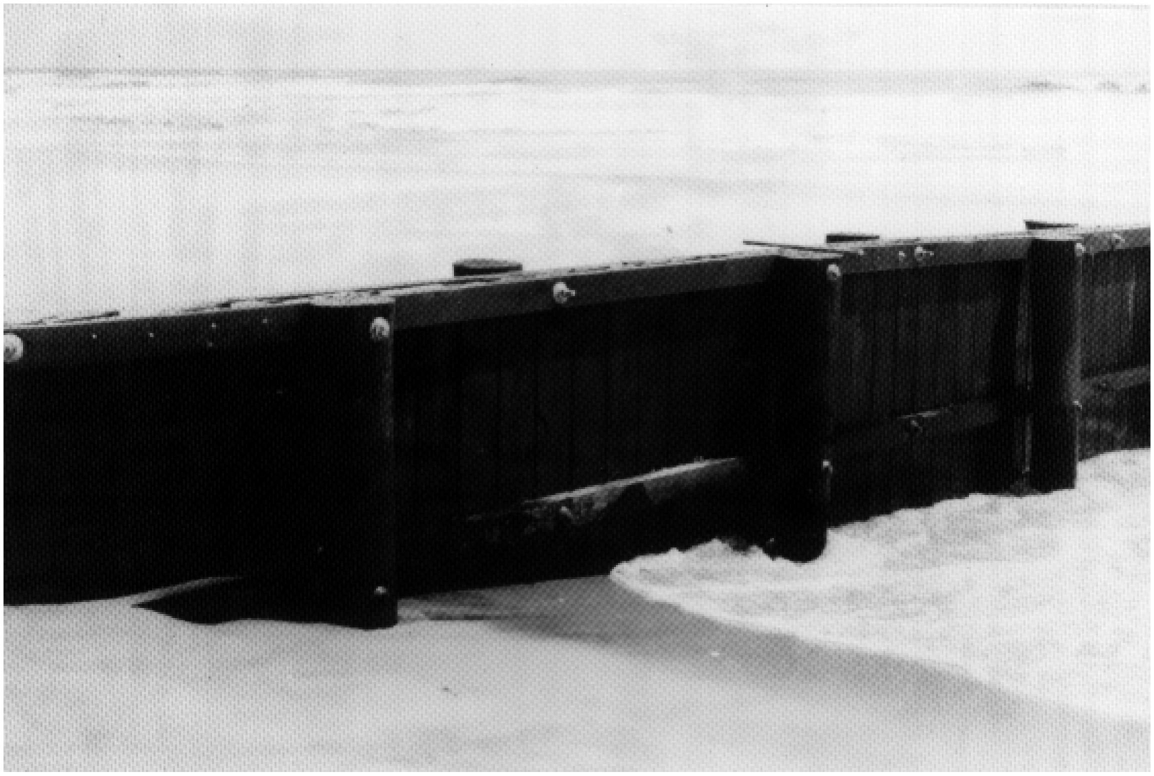
(c) Timber sheet-pile structures are often of ship-lap, tongue-and-groove, or Wakefield construction and are built of timber impregnated with creosote or some other preservative to slow deterioration and protect against marine borers. Overlapping timber sheet piles are usually jetted into the bottom, stiffened longitudinally by timber walers, and supported laterally by timber piles (Figure 2-2). Timber pile groins and bulkheads have been used extensively along ocean, Great Lakes, river, and

a. **Westhampton Beach, New York (1972)**

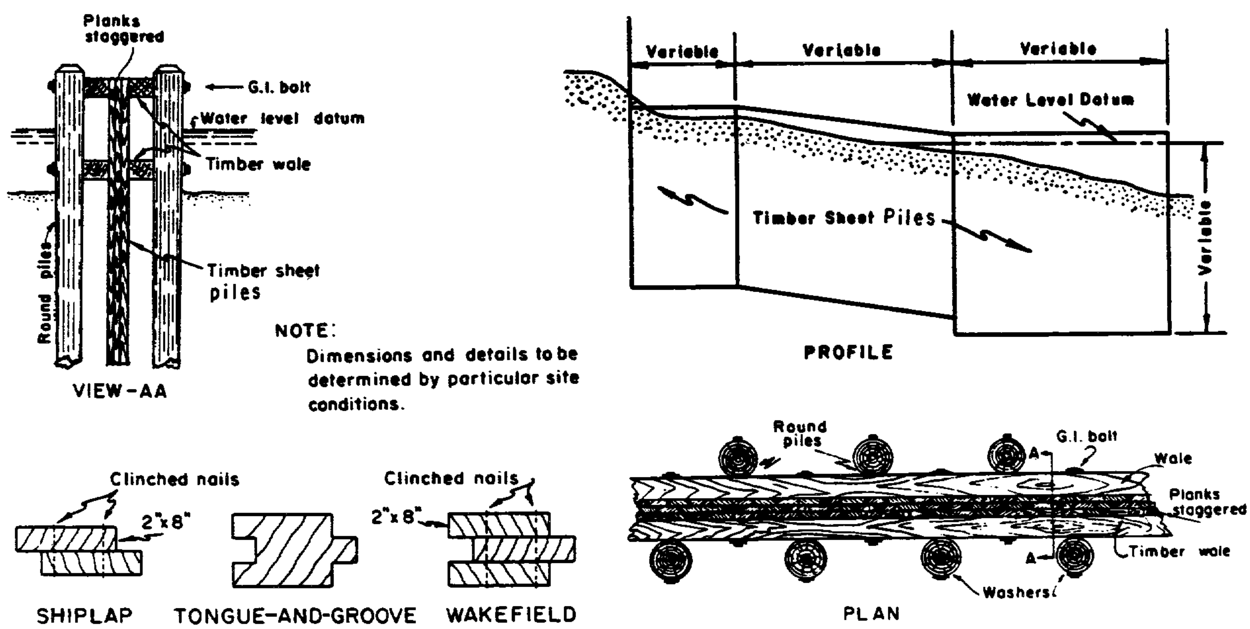


b. **Cross section**

Figure 2-1. Typical quarrystone rubble-mound groin



a. Wallops Island, Virginia (1964)



b. Cross section

Figure 2-2. Timber sheet-pile groin

estuary shorelines in the United States. Breakwaters and offshore sills built of timber sheet piles are less common.

(d) Properly designed concrete sheet-pile structures are more durable than structures built of other types of sheet piling. They are also usually more expensive. The dimensions of precast concrete sheet piles and the amount of reinforcing needed varies with the design. Lateral earth and wave forces usually establish critical design loads. Concrete sheet piles are designed with a key so that adjacent piles interlock. Longitudinal stiffness is usually provided by timber walers on both sides of the sheet piles fastened together with stainless steel bolts through holes precast into the piles or with a reinforced concrete cap (Figure 2-3). The concrete piles themselves usually provide lateral support or may be braced with tie-rods and piles. Groins, jetties, and bulkheads have all been built of concrete sheet piling.

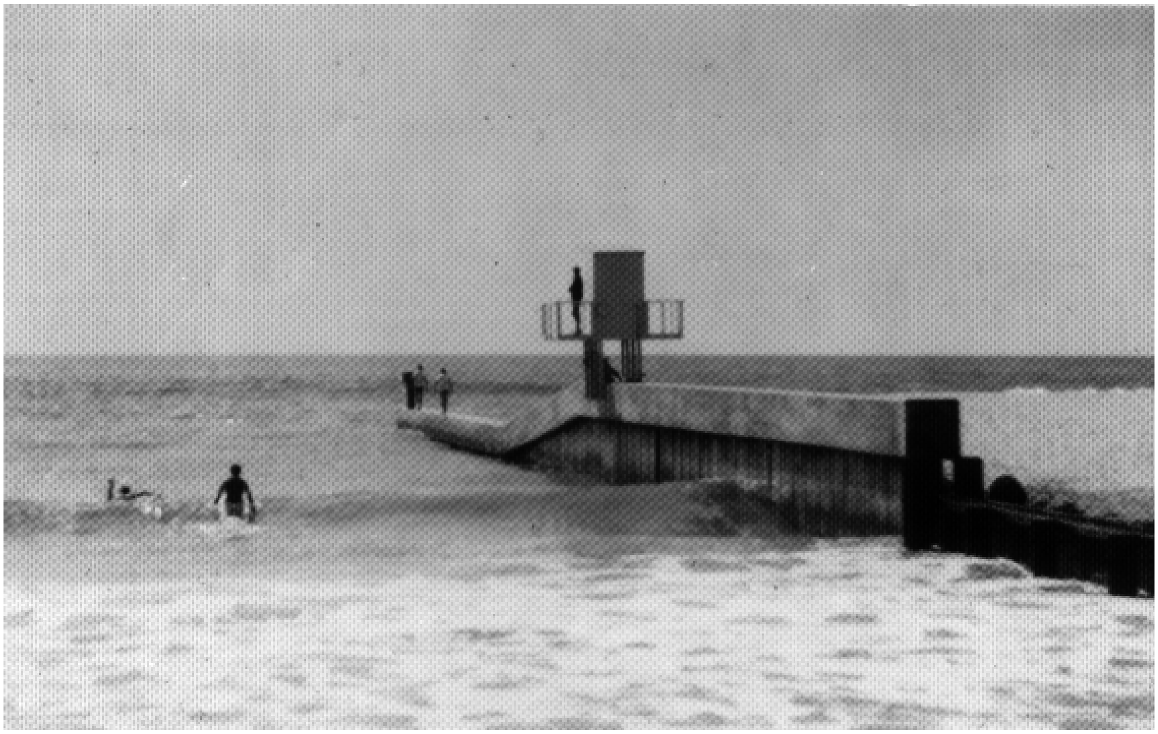
(e) Steel sheet piles are rolled structural shapes having various cross-sectional properties. The pile cross section, which may be straight, U-, or Z-shaped, has a channel along its edge that allows adjacent piles to interlock. Various section moduli are available to carry expected lateral earth and wave forces. Beach stabilization structures built of steel sheet piling are generally of two types: a single row of cantilevered piling with walers and often with adjacent piles to provide additional lateral support, and cellular structures. Structures built of a single row of piles are similar in design to the timber and concrete structures described. They are used primarily for bulkheads and low groins (Figures 2-4 and 2-5). Cellular structures are designed for large lateral loads. In plan, they consist of intersecting circular cells filled with earth, sand, or rubble and are then capped with rubble or concrete to contain the fill (Figure 2-6). Cellular sheet-pile structures have been used for both groins and offshore breakwaters, mostly in the Great Lakes.

(3) Other types of construction. Numerous other types of construction have been used for beach stabilization structures with varying degrees of success. For example, timber-crib structures have been used in the Great Lakes for breakwaters and jetties. These structures consist of a timber outer structure or crib into which rubble or stone is placed. This type of structure allows smaller stone to be used, which by itself would not normally be stable under wave attack. The timber crib allows the smaller stone to act as a unit. Gabions, wire baskets filled with stone, operate on the same principle but at a smaller scale (Figure 2-7). Gabions have been evaluated as low-cost shore protection, but are used primarily for stream bank or slope protection.

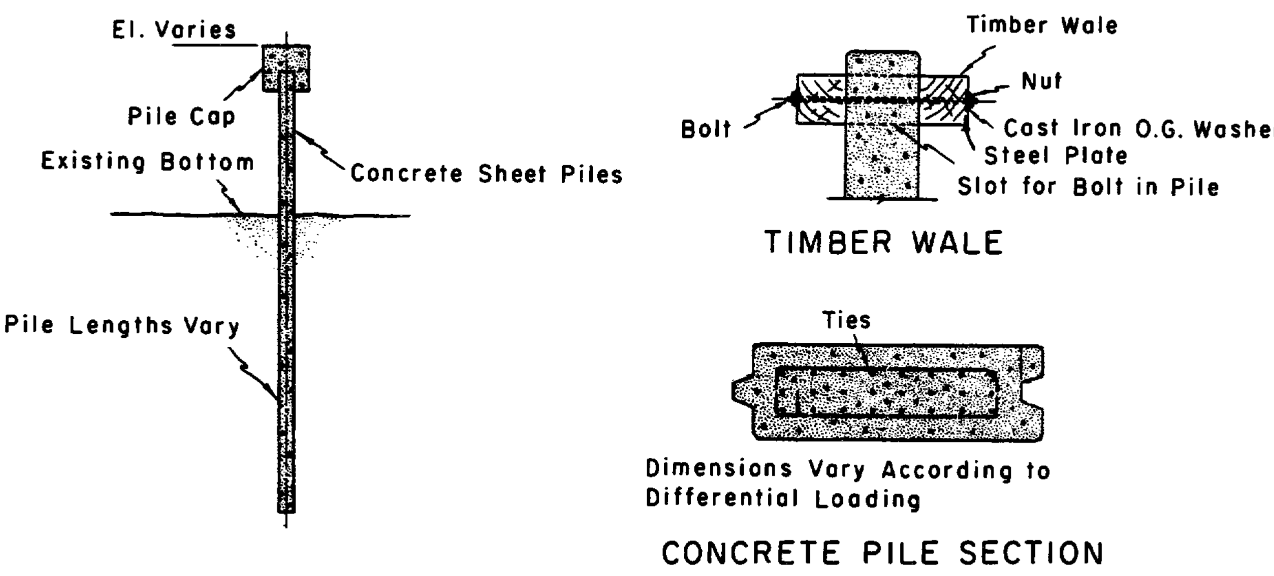
(4) Materials. Construction materials also impact on the effective service lifetime of beach stabilization structures. Timber structures that experience alternate wetting and drying, even those initially treated with wood preservatives, are subject to rotting whereas submerged portions are subject to marine borers when preservative protection deteriorates. Structural engineers should be consulted and involved in the selection of materials for beach stabilization structures. Determination of the best available material is dependent on many factors, such as expected project life, construction access, frequency, and accessibility of maintenance operations, and cost. These factors are considered in conjunction with the fact that these types of structures are located in severe, highly corrosive environments.

(a) Concrete. When reinforcement becomes exposed, especially in a saltwater environment, corrosion of the steel takes place causing cracking and spalling of the concrete. Methods of reducing this include: increasing concrete cover (concrete cover should be increased when designing structures for beach projects; proper consolidation is also critical to accomplishing this); use of epoxy-coated reinforcement, if necessary; and increasing the impermeability of the concrete. Retarding the ingress of chlorides and oxygen through the concrete is another method of reducing corrosion. This can be accomplished through the use of concrete mixes with low water/cement ratios. Type 2, sulfate-resistant cement should also be specified.

(b) Steel. Corrosion of steel members in coastal structures (which include piles, beams, channels, angles, tie-rods, and bolts) results in a loss of section that reduces the load-carrying capacity of the member. Selection of an appropriate protection system requires an assessment as to the feasibility (economically and logistically) of providing future maintenance. Coal tar epoxy is generally used in marine environments for protection of all members. Cathodic protection is another way to protect against corrosion; however, the cost of electricity and the replacement of sacrificial anodes increase operating costs. Aluminum and other metals may also react with seawater or soil. Abrasion of structural materials near the bottom by wave-agitated sand may also contribute to structural deterioration. In some cases, abrasion collars have been provided on structures at the sand line. Other conditions may prohibit the driving of steel piles, such as areas of hard, subsurface material and the existence of structures within the close proximity of driving operations.



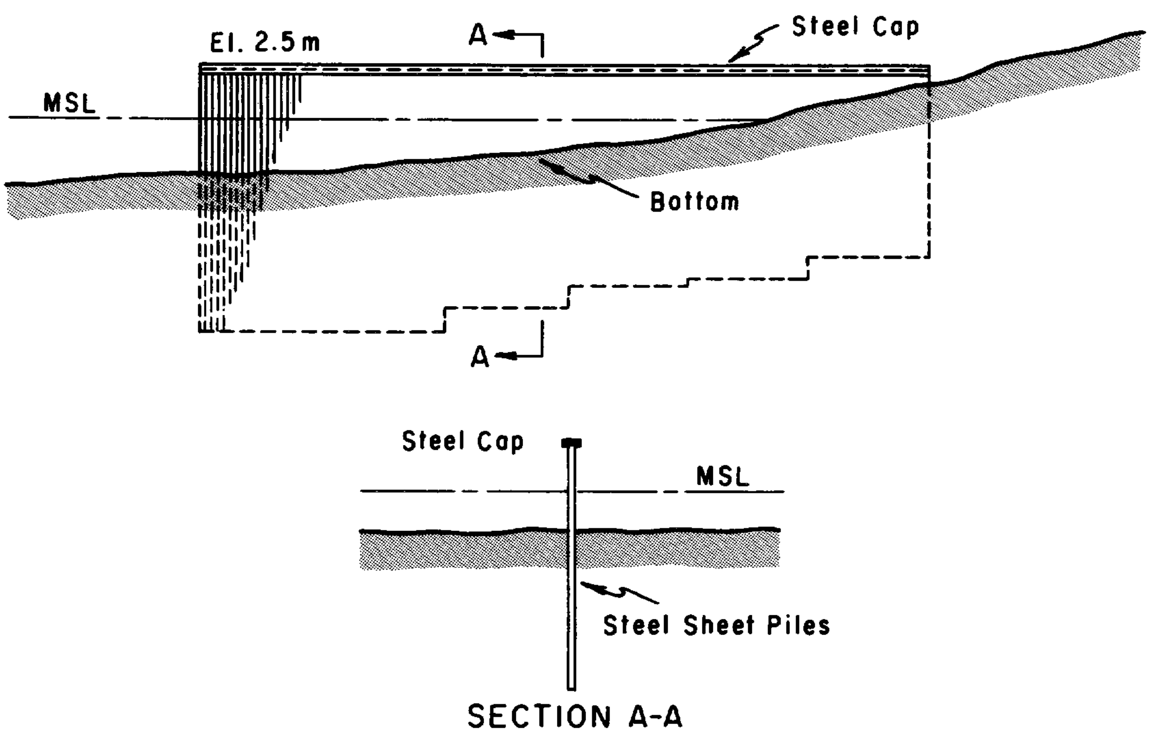
a. Doheny Beach State Park, California (October 1965)



b. Concrete pile section

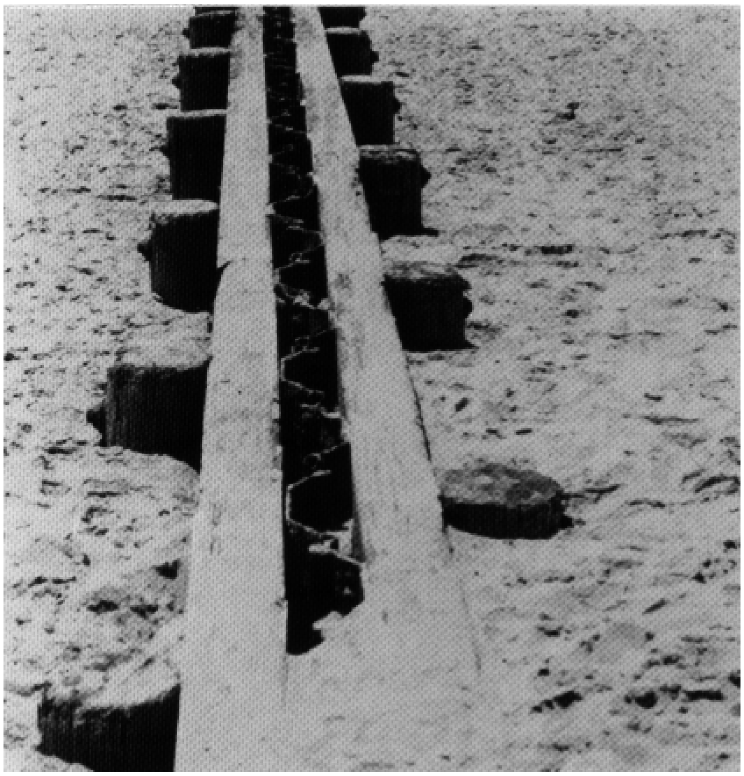
Figure 2-3. Cantieverted concrete sheet-pile structure with concrete cap (groin)

a. Newport Beach, California (March 1969)

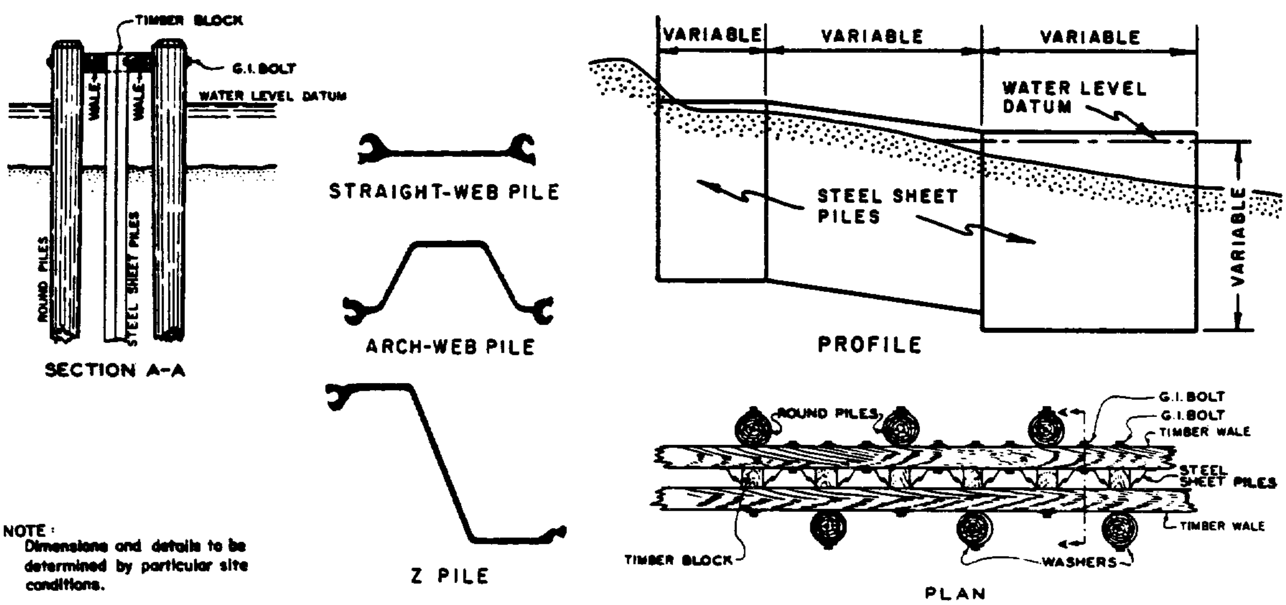


b. Cross section

Figure 2-4. Cantilevered steel sheet-pile structure with steel cap (groin)



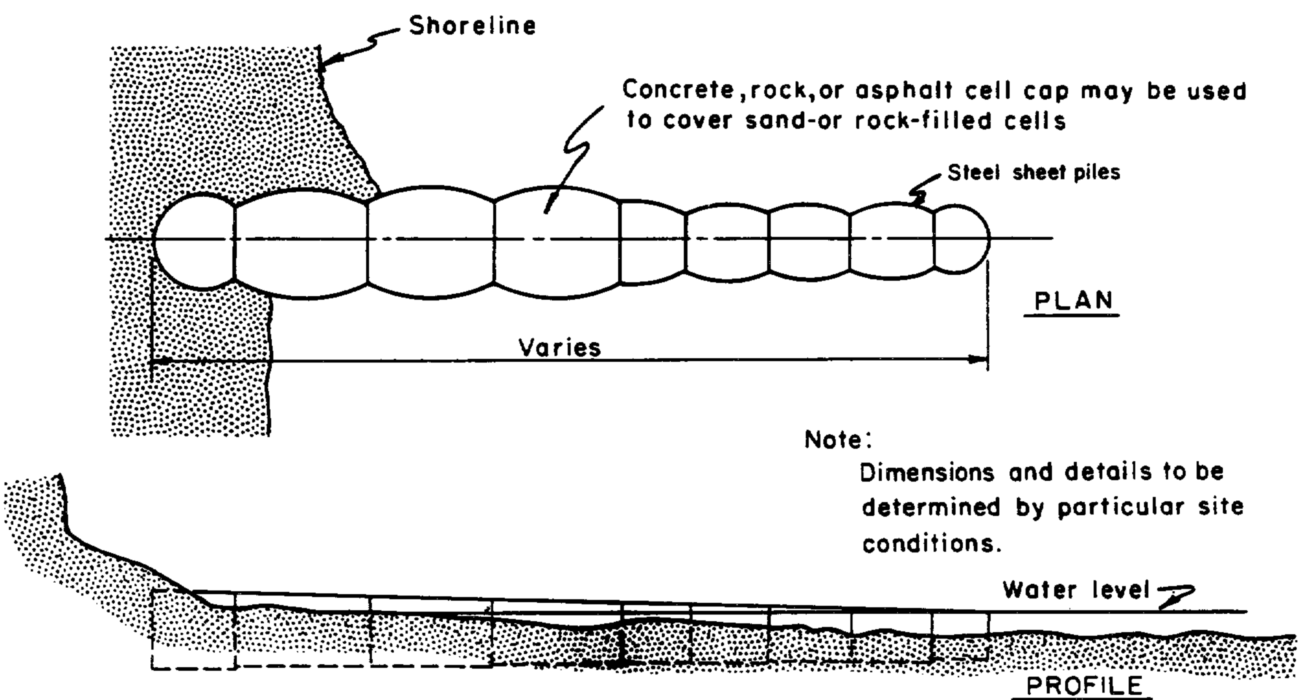
a. New Jersey (September 1962)



b. Cross section

Figure 2-5. Steel sheet-pile and timber wale structure (groin)

a. Presque Isle, Pennsylvania (October 1965)



b. Cross section

Figure 2-6. Cellular, steel sheet-pile structure (groin)

Figure 2-7. Gabion structure (revetments and groins)

e. Alternative beach stabilization methods.

(1) There are numerous proprietary beach erosion control and stabilization systems that function similar to groins, breakwaters, or submerged sills, but are of a unique geometry or type of construction. Most such structural systems are precast concrete units or flexible structures such as large sand-filled bags placed in various configurations on the beach or nearshore in shallow water. Most have undergone only limited field testing and many have never been field tested. Proponents of the various alternative schemes, usually the inventor or a vendor, often make unsubstantiated claims of success for their system. In fact, since they function either as groins, nearshore breakwaters, or perched beaches, they compete economically and functionally with traditional types of groin and breakwater construction such as rubble-mound and sheet-pile structures. The alternative structure systems, by themselves, do not increase the amount of sand available, but like their more traditional counterparts, redistribute available sand.

(2) Some of these structures have been evaluated under a program established by the Shoreline Erosion Control Demonstration Act, and their performance has been summarized by the Chief of Engineers in his report to Congress (Dunham et al. 1982). Field tests conducted under this program were all in sheltered US waters and not on the exposed ocean coast. Experience with most alternative beach stabilization systems on the open coast has been limited. In some cases, the results of experiments using open coast installations have not been reported because they have not been successful and, in some cases, successes have been selectively reported.

(3) One beach stabilization system, based on a different physical process, is beach-face dewatering. Under this system, a perforated drain pipe is installed beneath the beach face in the intertidal zone to lower the water table in the zone between low tide and the limit of wave runup at high tide on the beach (Figure 2-8). The lowered water table produces a ground-water hydraulic gradient in a direction opposite to that which normally prevails on a beach. This in turn results in the buildup of sand on the beach face. Presumably, sand in the water carried up the beach during wave uprush is not carried back offshore in the return surface flow, but rather, the altered groundwater gradient causes the surface flow to infiltrate into the sand, leaving the sand behind on the beach face. The result is an initial buildup of sand and stabilization of the beach face. Water collected by the perforated drain is carried to a collector pipe and then to a sump from which it is pumped back to the sea. The system thus requires drain and collector systems buried on the beach face and a sump and pumping system, which must be operated either continuously or periodically. Beach dewatering systems have been installed in Florida (Terchunian 1989), Namibia, and Denmark (Hanson 1986). Laboratory studies of beach dewatering systems have been conducted by Machemehl (1975) and Kawata and Tsuchiya (1986). Bruun (1989) and Parks (1989) also discuss beach dewatering.

(4) Evaluations of alternative beach stabilization systems should be based on their functional performance, their economics relative to traditional types of groin and breakwater construction, aesthetics, and their ability to be removed or modified if they do not function as expected or become aesthetically unacceptable. Since many systems are patented, they may also involve sole-source procurement or the payment of royalties to the inventor or licensee.

2-2. General Data Requirements for Design

a. Water levels.

(1) The range of possible water levels in the vicinity of a project is needed for both functional and structural design of beach stabilization structures. Prevailing water levels will determine where wave forces act on a structure and where the erosive action of waves will be felt on the beach profile. For example, during high-water levels, waves might attack the toe of a bluff that is normally above the active beach profile.

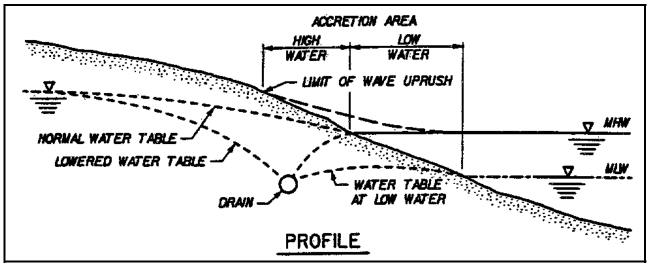


Figure 2-8. Beach dewatering system--lowered beach water table on beachface

(2) Many coastal structures extend across the surf zone so that different elements of the structure are subected to critical design conditions at different water levels. Thus, designs should not ordinarily be based on a single design water level, but rather on a range of reasonably possible water levels. For example, at low water the seaward end of a groin might experience breaking waves while more landward sections of the groin experience broken waves. At higher water levels, a more landward section of the groin might experience breaking waves, and the seaward end will experience nonbreaking waves. Sometimes the stability of a rubble structure depends critically on the water level at the toe of the structure since the stability coefficient depends on whether the waves are breaking or nonbreaking waves. The location on a structure where a wave of given height and period breaks depends on water depth and nearshore slope; hence, there will often be a critical water level where maximum wave effects (minimum structure stability or maximum forces) occur. Design calculations should recognize this factor, and a reasonable range of water depths should be investigated.

(3) Data on the range of water levels expected at a breakwater site are needed to determine the variation in a breakwater's distance from shore. During high-water levels, a breakwater will be farther from shore than during low-water levels. Some nearshore breakwaters have been observed to have significantly different low-water shorelines than high-water shorelines. For example, at Winthrop Beach, MA, a tombolo is exposed at low tide while only a salient is present at high tide (Figure 2-9). Wave conditions in the lee may be affected by prevailing water levels. Also, as water levels increase, freeboard is reduced, and wave overtopping of the breakwater may occur. Statistical data on water levels and the resulting breakwater freeboard establish the frequency of wave overtopping, a factor that influences the shape of the shoreline behind the structure. Frequent overtopping can prevent the formation of a tombolo and may also result in currents through the gaps in multiple breakwater systems. Surf zone width may also change the area where longshore transport occurs relative to the breakwater.

(4) Because water level changes are caused by astronomical tides, storm tides, and in the case of the Great Lakes, long-period hydrologic factors, water levels are usually described statistically. The frequency, or probability that a given water level will be equaled or exceeded, or its return period in years (the reciprocal of the probability of exceedence) is defined (Figure 2-10). Thus, for example, the water level that is exceeded on average once in 100 years (a probability of 1/100 = 0.01 of being exceeded in any 1 year) might be specified as a design water level. Significant deviations from predicted astronomical tidal levels will occur during storms because of meteorological tides (storm surges) caused by strong onshore winds and low atmospheric pressure. Consequently, design water levels for a structure may include a storm surge with a specified return period. The statistics of meteorological tides are usually based on recorded water levels at tide gaging sites or joint probability analysis of storm parameters and predicted surge heights.

(5) Water level data for coastal sites are often available from Corps of Engineers' General Design Memoranda for coastal sites where earlier studies have been conducted, Federal Emergency Management Agency (FEMA) flood insurance studies, or the National Oceanic and Atmospheric Administration's (NOAA's) National Ocean Service (NOS) for areas where NOAA operates tide gages. The location of NOAA's principal tide measuring stations along with the period of record are given in the annual NOAA "Tide Tables" publication (for example, see NOS 1986). Data on historical water levels of the Great Lakes and lake level statistics are available from NOS (1986) and from the US Army Engineer District (USAED), Detroit (for example, USAED 1986). Water level statistics for the US East Coast are given by Ebersole (1982). Water level statistics for predicted astronomical tides are also given by Harris (1981). This statistical compilation provides information on the fraction of time that water levels will be above a given level at a site (Figure 2-11).

(6) Studies by the National Academy of Sciences (Charney et al. 1979, Dean et al. 1987) and the Environmental Protection Agency (Hoffman 1984, Barth and Titus 1984) indicate that the rate at which sea level is rising may increase in many areas of the world as the possible result of a general global warming trend. Past rates of sea level rise (where sea level has been rising)

a. Low tide

b. High tide

Figure 2-9. Breakwater at Winthrop Beach, MA 1981) (Dally and Pope 1986)

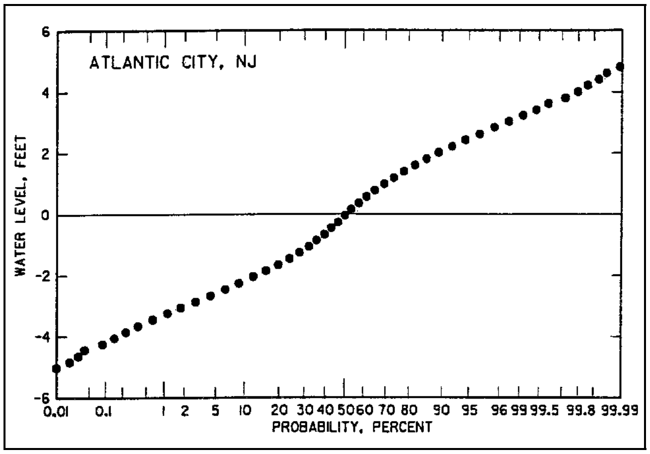


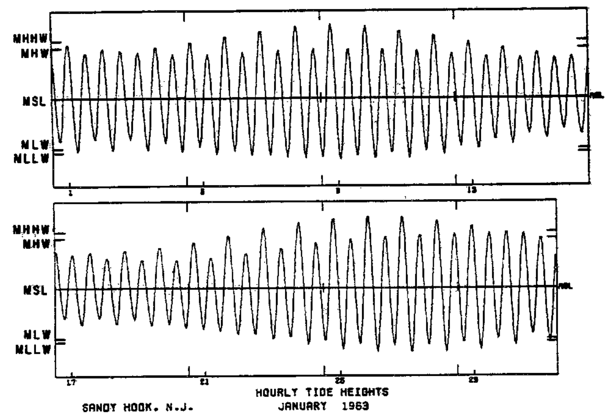
Figure 2-10. Statistical distribution of annual net longshore transport rates. (To convert feet into meters, multiply by 0.3048)

have been less than 1 foot (0.3048 meter)/century. The rate of relative sea level rise will vary with geographic location because it is influenced by local land subsidence or rebound. Data on local US experience with relative sea level change are summarized in Hicks (1973) and Hicks et al. (1983). Projection of past historic relative sea level change should be used in project design. Long-term erosion rates have been correlated with increases in local mean sea or lake level (Bruun 1962, Hands 1981). Procedures to calculate long-term erosion rates attributable to a rise in water level are given in Bruun (1961) and Weggel (1979). If the rate of relative sea level rise changes, the rate of erosion will likewise change. Prudence may require an allowance in a project design for the continuation over the project design life of an established significant long-term trend in relative sea level rise. Consideration must be given to the confidence band of the data the designer is using, the tolerance allowed in constructing the project, and whether it is more cost effective to include the allowance for the significant sea level rise in the initial construction or to plan for modification later, after the need for such is demonstrated.

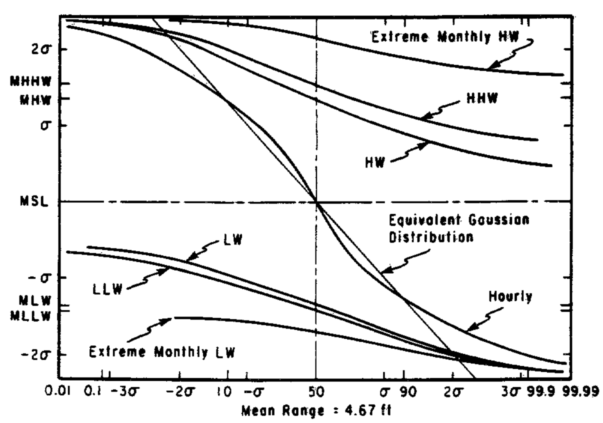
b. Waves.

(1) Wave data are needed for both structural and functional design of beach stabilization projects. Waves generally cause critical design forces on coastal structures. Waves also transport sediments onshore, offshore, and alongshore and therefore can transport sediments into and out of a project area as well as redistribute it within an area.

(2) Wave data required for structural design differ from data needed for functional design. For structural design, a characteristic wave height associated with a given frequency of occurrence or return period is usually needed. Thus, for example, the significant or root-mean-squared (rms) wave height that is exceeded on average once in 50 years or once in 100 years might be chosen for



a. Hourly tide heights



b.Comparison of water levels

Figure 2-11. Statistics of predicted astronomical water levels (Harris 1981). (To convert feet into meters, multiply by 0.3048)

design. The largest probable wave for the given sea state and storm duration might then be selected for the structural design, or a lower wave in the spectrum (such as the 10-percent wave or the significant wave) might be used if a flexible structure such as a rubble-mound groin or breakwater is being designed. Ultimately, the selection of a design wave should be based on an evaluation of the consequences of a structural failure, both the public safety and economic consequences. Structural design, therefore, focuses on the larger waves in the wave climate at a site since large waves generally result in critical design conditions.

(3) For functional design, a more complete wave data record is needed because sediment can move under even relatively small waves. The time series of wave height, period, and direction is needed to estimate the amount of sediment in transport alongshore. Net and gross transport rates are usually the summation of daily transport rates computed using Method 3 outlined in the SPM (1984). The SPM equation for estimating longshore transport rates requires knowledge of a characteristic wave height (usually the significant height), a characteristic wave period (usually the period of maximum energy density in the wave spectrum), and wave direction relative to the trend of the shoreline.

(4) For functional design of breakwaters, wave heights, periods, and directions are needed primarily to determine longshore sand transport rates. Incident wave heights, periods, and directions also determine wave conditions in the lee of a nearshore breakwater and establish the shape of the shoreline. The shoreline that evolves behind the structure depends on the range of wave heights and directions at the site and their seasonal variability.

(5) For groin design, wave height statistics and water levels are needed to determine the level of wave action to which various portions of a groin will be subjected. Because of its nearshore location, waves along the shoreward portion of the groin will be depth limited, i.e., maximum wave heights depend on water depth, wave period, and beach slope as given in Figure 2-12. Waves may or may not be depth limited at the seaward end of a groin depending on the prevailing water depth and on the height of incoming waves. Figure 2-12 can be used to determine the water depth seaward of which waves are no longer depth limited if the local height of the incoming waves is given as a function of water depth (Figure 2-13). For wave force and rubble-mound stability computations, design wave conditions with a given return period are usually specified, e.g., wave conditions with a return period of 20 or 50 years might be specified as the design wave height.

(6) Wave height statistics to determine design conditions will normally be based on hindcast wave data because a relatively long record is needed to confidently extrapolate the data. Wave gage records rarely cover a sufficient number of years to permit extrapolation. Corson and Tracy (1985) present extremal wave height estimates for 73 Phase II Stations of the Wave Information Study (WIS) Atlantic coast hindcasts. Also, Phase III WIS data for nearshore locations (Jensen 1983) can be plotted on extremal Type I (Gumbel) probability paper and extrapolated to longer return periods. Figure 2-14 is a plot of annual maximum wave heights ranked by height as a function of return period determined from the Weibull plotting position formula:

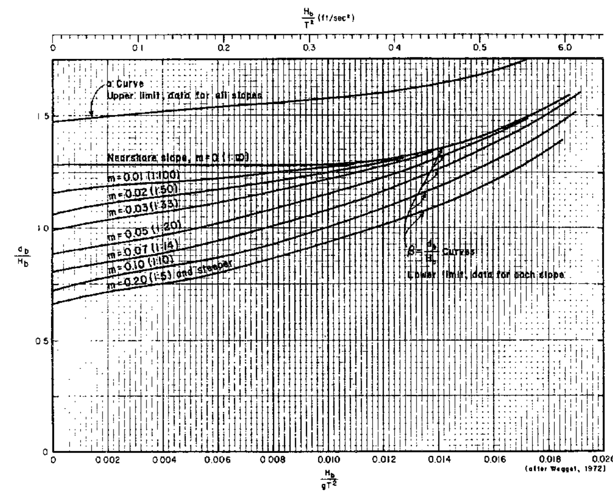


Figure 2-12. Water-depth-to-wave-height ratio at breaking as a function of wave steepness and beach slope (after Weggel 1972)

$$T_R = \frac{N + 1}{m} \qquad (2\text{-}1)$$

where

T_R = return period in years
N = number of years of record
m = rank of the given wave height (m = 1 for the largest annual wave height
m = 2 for the second largest, etc.)

(7) The prevailing wave direction will determine the shoreline orientation. The shoreline will move to orient itself more nearly parallel with incoming wave crests. If waves approach a beach from a predominant direction during one season, in time the shoreline will shift until it is parallel with the incoming waves of that season. When the direction of wave approach changes, the shoreline will eventually shift in response to the change if the wave conditions persist. For example, if the direction of

incoming waves changes for a period of time, the fillet in a compartment between two groins may shift from one groin to the other. The amount of sand in the groin compartment is usually assumed to be conserved so that if the wave directions are known, the shoreline response can be determined once the profile shape is known. The best indicator of prevailing wave direction is the shoreline orientation at nearby groins.

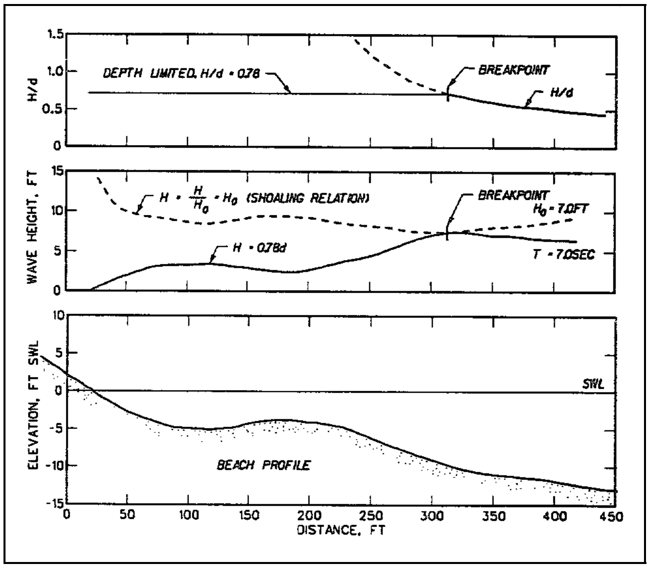


Figure 2-13. Wave height as a function of water depth and bathymetry, shoaling wave over irregular beach profile

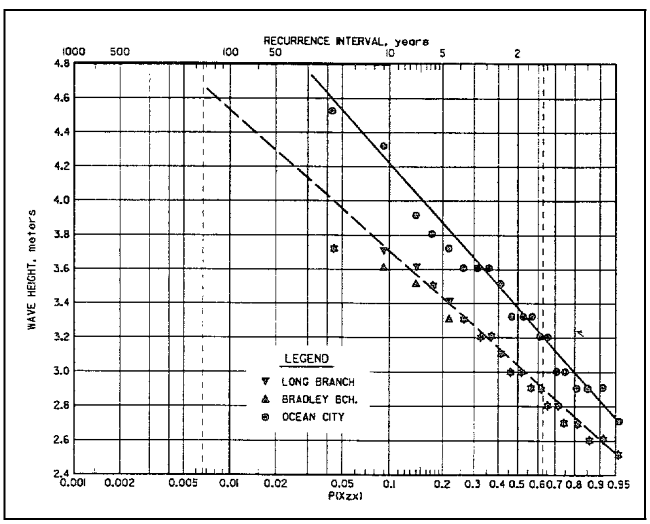


Figure 2-14. Annual maximum wave heights as a function of return period, Long Branch, Bradley Beach, and Ocean City, NJ

(8) Application of wave and water level data to predicting onshore/offshore transport rates is not well developed, although in recent years several beach profile evolution models have been developed (Swart 1974, Kriebel 1982, Hughes 1983, Kriebel and Dean 1985). In addition, several models for beach profile and dune response to storms are available (Edelman 1968, Edelman 1972, Moore 1982, Vellinga 1983, Larson et al. 1990). Generally, the beach profile shape and its evolution depend on wave height, water level, wave-height-to-wave-length ratio (wave steepness), antecedent wave and beach profile conditions, and sediment characteristics such as mean grain size, grain size distribution, and grain shape. Wave conditions and water levels prevailing during both typical and extreme storms in a coastal area may be needed to evaluate the performance of a particular beach and dune profile and any associated beach stabilization structures. Additional guidance on water levels and wave heights for coastal design is provided in EM 1110-2-1412 and EM 1110-2-1414.

c. Longshore sand transport rates.

(1) Longshore transport is the most significant process for moving sediments in the coastal zone. Information on prevailing longshore sand transport rates is needed for the planning and design of all beach stabilization projects. The longshore sand transport rate, Q, is a measure of the rate at which littoral material moves alongshore in the surf zone from currents produced by obliquely breaking waves. These transport rates are needed to perform sediment budget calculations for an area, determine the amount of sand naturally available to fill groins or offshore breakwaters, determine whether beach fill is necessary for a project, and estimate how much sand will bypass a project to nourish downdrift beaches. Pre- and postproject sediment budgets should be developed for both the immediate project area and the adjacent shorelines.

(2) Longshore sand transport rates are usually specified as annual rates. The annual net transport rate is the net amount of sediment moving past a point on the beach in a year. Mathematically, it is given by:

$$Q_n = \frac{1}{T} \int_{t}^{t-T} Q(t)\ dt \qquad (2\text{-}2)$$

where

Q_n = net longshore sediment transport rate

T = time period over which the transport rate is averaged (usually 1 year)

t = time

$Q(t)$ = instantaneous longshore transport rate (positive or negative depending on whether transport is to the right or left for an observer looking seaward)

(3) The annual gross transport is the total amount of sediment moving past a point regardless of the direction in which it is moving. Mathematically, it is given by:

$$Q_g = \frac{1}{T} \int_{t}^{t-T} |Q(t)| dt \tag{2-3}$$

(4) The net and gross transport rates in terms of the positive and negative rates are given by:

$$Q_n = Q(+) - Q(-) \tag{2-4}$$

and

$$Q_g = Q(+) + Q(-) \tag{2-5}$$

where

$Q(+)$ = cumulative annual positive transport (total transport to the right per year for an observer looking seaward)

$Q(-)$ = cumulative annual negative transport (total annual transport to the left)

For the sign convention adopted, $Q(-)$, $Q(+)$, and Q_g are always positive, and Q_n may be either positive or negative.

(5) Therefore, the annual positive and negative transports are given by,

$$Q(+) = \frac{1}{2} (Q_g + Q_n) \tag{2-6}$$

and

$$Q(-) = \frac{1}{2} (Q_g - Q_n) \tag{2-7}$$

(6) The SPM (1984) suggests four ways of deriving longshore sand transport rates at a site. Method 1 recommends adoption of the best-known transport rate from a nearby site making appropriate adjustments if necessary to account for differences in exposure, sheltering, shoreline alignment, etc.

(7) Method 2 relies on documented sediment accumulations or shoreline changes in the vicinity of spits, inlets, or coastal structures. The volume of sediment accumulated in the time between two topographic/bathymetric surveys of the site is divided by the time between surveys to estimate the average rate of accumulation. Transport rates found in this way may approximate either the net or gross transport depending upon the process causing the accumulation. If based on accumulation at a spit, an estimate of net transport is obtained; if based on accumulation in an inlet, an estimate of gross transport is obtained. The basic principle involved in applying this method is to construct a simple sediment budget for a section of shoreline (or inlet) with the assumption that the influx and/or efflux of sediment is known at some location. At a spit, for example, the efflux at the distal end of the spit is assumed to be zero, and the net volume of sediment transported alongshore onto the spit accumulates there. (Changes in shoreline orientation along the spit and the resulting variations in longshore transport are generally ignored. This leads to some error.) For an inlet, sediment entering the inlet by longshore transport from either side of the inlet is assumed to be trapped, and the natural efflux of sediment from the inlet is zero. Thus, the gross longshore transport is estimated. Inlet dredging must be accounted for in determining the volume of sediment trapped. Any sediment naturally bypassing the inlet results in underestimating the gross transport.

(8) Method 3 is based on the assumption that the longshore transport rate, Q, depends on the longshore component of energy flux in the surf zone. The "Coastal Engineering Research Center (CERC) formula" (Equation 4-49, SPM 1984) for estimating the potential longshore transport rate is given by:

$$Q = \frac{K}{(\rho_s - \rho)\, g\, a'} P_{ls} \qquad (2\text{-}8)$$

where

K	= dimensionless empirical coefficient
ρs	= sediment density
ρ =	water density
g	= acceleration of gravity
a'	= solids fraction of the in situ sediment deposit (1 - porosity)

$$P_{ls} = \frac{\rho g}{16} H_{sb}^2\, C_{gb} \sin(2\, \Theta_b) \qquad (2\text{-}9)$$

where

H_{sb}	= nearshore breaking height of the significant wave
C_{gb}	= wave group speed at breaking
Θ_b	= angle breaking wave crest makes with the shoreline

In shallow water,

$$C_{gb} = \sqrt{g d_b} \qquad (2\text{-}10)$$

where d_b is the water depth at breaking, usually assumed to be linearly related to the breaking wave height as,

$$H_b = \gamma\, d_b \qquad (2\text{-}11)$$

where the breaking wave index, γ, is equal to 0.78.

(9) Equation 2-8 provides an estimate of the longshore transport rate in terms of breaking wave parameters. Wave data estimates may be obtained through Littoral Environment Observation (LEO) data (Schneider 1981) or by transforming waves inshore to breaking from an offshore source such as a wave gage or WIS data. The effect on a project of daily and seasonal variations in transport conditions can be studied when variations in wave conditions are known. For example, wave height, period, and direction data available from WIS wave hindcasts may be used to estimate a typical time series of longshore transport. The SPM (1984) provides a more detailed explanation of the equations and assumptions used in Method 3. Computation of longshore flux using LEO data is discussed in Walton (1980).

(10) Method 4 provides an empirical estimate of the annual gross longshore transport rate, which is also an upper bound to the annual net transport rate. A variation of the equation developed by Galvin (1972) is given by:

$$Q_g = 0.03636 \sqrt{g}\; H_b^{5/2} \qquad (2\text{-}12)$$

where

Q_g = annual gross transport at a site
g = acceleration of gravity
H_b = average annual breaker height at the site

The average breaker height can be obtained by averaging visual observations such as those obtained under the LEO Program, WIS, or gage data. Equation 2-12 is dimensionally consistent.

(11) Another approach for examining longshore transport develops a sediment budget based on estimates of inputs including bluff recession and stream sediment contributions. This method is commonly used along the Great Lakes and part of the Pacific coast, since Equation 2-8 can greatly overestimate transport in areas deficient of littoral material. The potential littoral transport rates *Q(+)* and *Q(-)* are determined from respective wave energy. The concept of littoral cells is applied; that is, a cell consisting of a self-contained stretch of coastline with its own sand sources, losses or sinks, and littoral drift connecting the two. Losses include offshore channels, canyons, sand mining, etc.

(12) Longshore transport rates may vary significantly from year to year, making it necessary to incorporate flexibility into the design of any shore protection project. For example, the net transport at a site might be in one direction one year and in the other direction another year. Gross transport rates exhibit similar variability with large gross rates occurring during particularly stormy years and lower gross rates in relatively calm years. Figure 2-15 illustrates the variability of annual net transport rates calculated from the WIS data for a site along the North Atlantic coast. This figure suggests that annual net longshore transport rates may be described by a Gaussian or normal probability distribution. The mean of the resulting distribution is the long-term average net longshore transport rate. The standard deviation of the distribution provides some measure of the annual variation

of the net longshore transport rate. The example distribution in Figure 2-15 shows that, on average, a year in which net transport is opposite to the long-term direction can be expected about once in 12.5 years for this site.

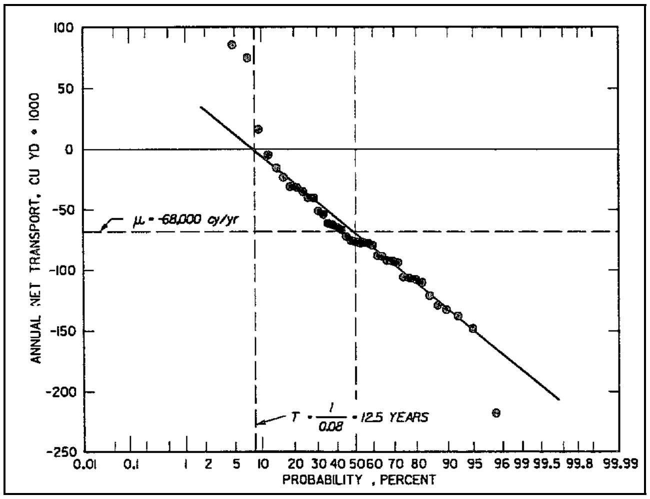
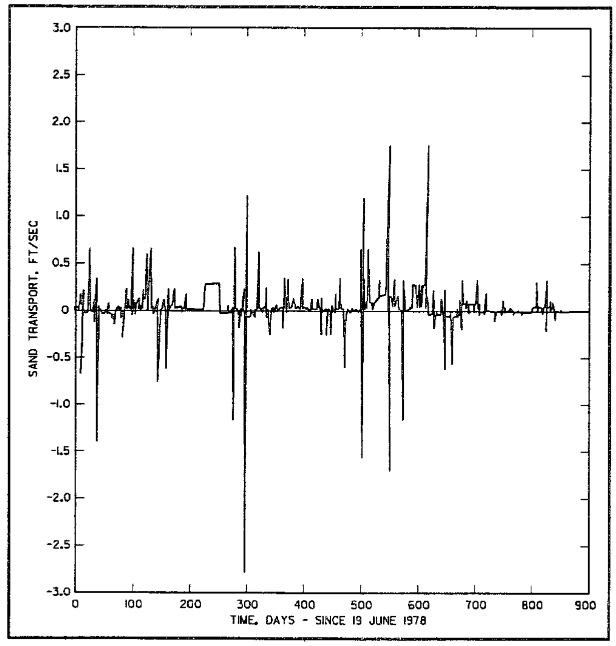


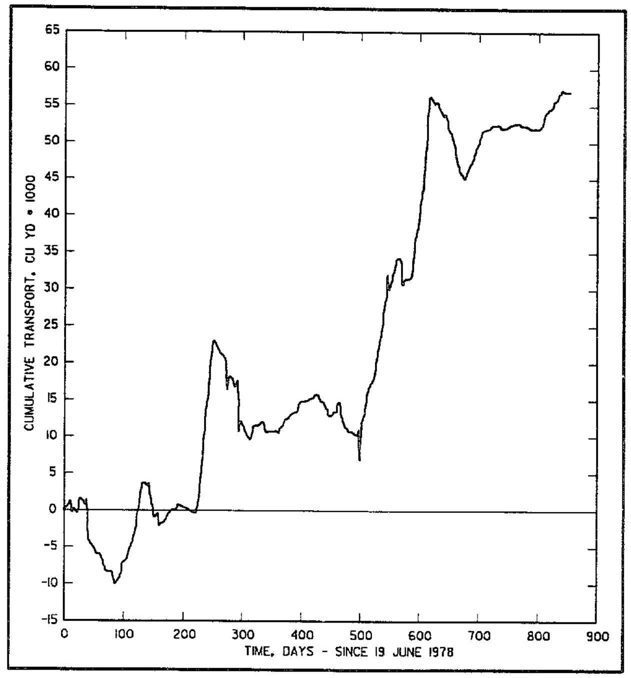
Figure 2-15. Statistical distribution of annual net longshore transportation rates. (To convert cubic yards to cubic meters, multiply by 0.76455)

(13) Longshore transport rates also vary seasonally. For example, along most reaches of the US Atlantic coast, net transport is southward during the winter months because of a relatively few intense "northeasters" that dominate the transport environment. These "northeasters" transport large volumes of sediment southward. During the late spring and summer months, net transport is northward because dominant waves are out of the southeast. Northward transport is usually smaller due to the lower wave heights generated during the spring and summer seasons. In response to the seasonal variations in transport direction, sand accumulation in the fillets adjacent to groins or behind nearshore breakwaters may move from one side to the other in response to prevailing transport conditions.

(14) Estimates of positive, negative, net, and gross longshore sand transport rates can be calculated from a wave climatology that includes wave heights, periods, and directions. Usually, the positive and negative (or the net and gross) transport rates will suffice for beach stabilization design. However, a time series of wave heights, periods, and directions permit the time series of longshore sand transport rates to be calculated. Figure 2-16(a) represents such a time series computed from daily visual wave observations. Figure 2-16(b), which is based on the data in Figure 2-16(a), is a plot of the cumulative amount



a. Time series



b. Cumulative longshore sand transport

Figure 2-16. Longshore sand transport, Slaughter Beach, Delaware

of sediment passing a point on the beach. With the development and improvement of computer models to simulate the evolution of shoreline changes near groins and breakwaters (Hanson and Kraus 1989).

d. Offshore bathymetry.

(1) Information on offshore bathymetry at a beach project site is needed for several purposes. If offshore structures or structures that extend seaward from the shore are being considered, bathymetric data are needed to establish the water depth at the site. This information will influence what type of shore protection is indicated, the wave and current forces to which they will be subjected, and the quantity of materials needed to build the structures. Offshore bathymetry is also important in the transformation of waves as they move from deep water toward shore. Wave refraction, shoaling, and diffraction by bathymetry alter local wave heights and directions. Locating potential sources of beach fill, such as offshore sand deposits and sand deposits in tidal inlets, also requires bathymetric surveys.

(2) Two bathymetric surveys of the same site spaced in time may be used to establish areas of accretion and erosion and to estimate erosion and accretion rates. The season when the two surveys were taken should be the same to distinguish long-term from seasonal changes. Bathymetric data can document the effect of structures on the offshore bathymetry and/or establish accretion/erosion patterns and rates in tidal inlets. Such accretion/erosion rates are needed to make sediment budget calculations and determine where and how much sand is available within an inlet for beach nourishment. More detailed analyses can also look at the patterns of erosion and deposition and the water depths in which these processes occur (Weggel 1983a).

(3) Approximate bathymetry for US coastal areas is given on US Geological Survey (USGS) 7.5-minute quadrangle topographic maps (quad sheets). However, bathymetry is continually changing, especially nearshore and in the vicinity of tidal inlets, capes, and river mouths, and these data may not be up-to-date. Naval Hydrographic Office charts also provide bathymetric data; however, they are intended primarily for navigation, and the bathymetry shown for shallow coastal areas away from established navigation channels may not be current. More recent and detailed bathymetric data may be available from the NOS in digital form or in the form of "boat sheets," raw data from which the bathymetry on USGS quad sheets is extracted. The preceding bathymetric data are often suitable for preliminary design or for wave transformation studies of areas distant from shore where bathymetric changes are less likely to occur. If up-to-date bathymetry is needed for project design or for documenting shoaling/erosion, it must usually be obtained during design. Special bathymetric surveys must be conducted if shore protection structures will extend offshore or if beach fill from offshore or inlet sources will be part of a project.

e. Shoreline changes. Measurements of shoreline changes are needed to establish short- and long-term erosion rates, determine typical and extreme seasonal movements of the shoreline, and determine the subaerial and subaqueous profile shape and its response to changes of wave conditions. Shoreline change data (both historical data and data obtained for a specific project's design) include profile surveys, aerial photographs, and other records documenting beach changes.

(1) Beach profiles.

(a) Periodic beach profile measurements that give the beach elevation along a line perpendicular to shore and extending offshore provide the most detailed information on shoreline changes; however, historical data may not be available for a given project site. Once a project is conceived and planning begins, a program of beach profile surveys should be initiated to acquire the needed data. Usually several years of such data are required. Profile data obtained during various seasons of the year are needed to establish normal and extreme seasonal shoreline movement and profile elevation changes. Storms usually occur more frequently during the fall or winter months when high, short-period waves result in "winter" or "storm profiles"; low, long-period, beach-building waves occur more frequently in summer resulting in "summer profiles" and wide beaches. In the Great Lakes, profiles respond to the seasonal rise and fall of the mean lake levels as well as to more long-period trends in water levels.

(b) If a groin is to serve as a template for the updrift postproject beach, the range of typical beach profile conditions at the site is needed to help establish the groin profile. The length of a groin is established by the expected beach profile adjacent to it and the desired location of the shoreline. The postproject profile is usually assumed to have a shape similar to the preproject profile; however, following construction, the profile on the updrift side of a groin will generally be steeper than the profile on the downdrift side (Figure 2-17). The difference in beach profile elevation between the updrift and downdrift sides of a groin will determine the lateral earth forces experienced by a sheet-pile groin and, since water depth

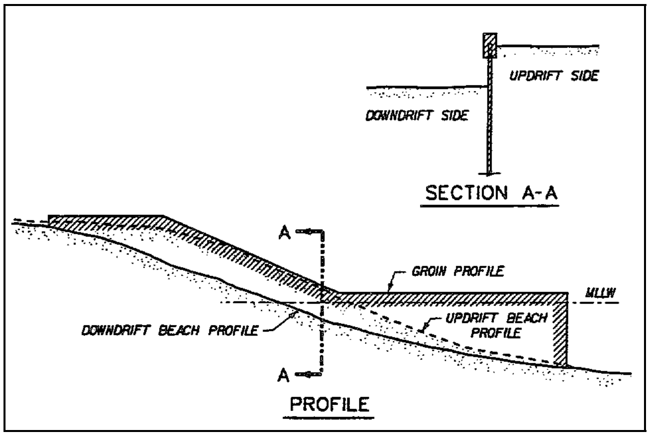


Figure 2-17. Groin profile showing differences in beach profile on updrift and downdrift sides

controls wave height in shallow water, the profile controls maximum lateral wave forces on a groin. Profile changes caused by scour adjacent to a groin must also be considered. Data on seasonal onshore-offshore profile movement are needed to determine the range of possible profile conditions on both sides of the groin. During periods when the groin is full and sand has built up against the updrift side, the profile determines how much sand will be transported over the groin on the beach face. A procedure for estimating shoreface transport rates over low groins and jetties is given by Weggel and Vitale (1985).

(c) Beach profiles can also provide data on the closure depth, the water depth beyond which there is no significant sediment movement (Weggel 1979, Hallermeier 1983). The closure depth plus the berm height gives an estimate of the beach area produced per unit volume of beach fill. For example, a closure depth of 27 feet with a berm height of 10 feet requires 27 + 10 = 37 cubic feet of sand to produce 1 square foot of beach. Even if beach fill is not part of a groin project, beach profiles and the closure depth are needed to compute sand volumes involved in beach alignment changes.

(d) Beach profiles at a nearshore breakwater project site are needed to determine the breakwater's location relative to the postproject shoreline and to estimate the volume of sand that will accumulate behind the breakwater. Except for in the immediate vicinity of the structure, profiles seaward of the breakwater can be assumed similar to preproject profiles. If beach fill is included in the project, the postproject profile will eventually be displaced seaward a distance approximately equal to the volume of fill per unit length of beach divided by the sum of the berm height and closure depth. The rate at which this seaward movement of the profile occurs is related to the rate at which the fill is distributed across the profile by wave action. This will occur more slowly for a nearshore breakwater project than for a beach fill without breakwaters. Beach profiles behind nearshore breakwaters will be steeper than preproject profiles. Preproject profiles will have to be adjusted using judgment in conjunction with any prototype data from similar breakwater sites to estimate how the postproject profiles will appear after construction.

(e) Offshore sills introduce a discontinuity into the nearshore beach profile. Preproject profiles can be used to estimate the postproject profile by shifting the preproject profile upward at the sill location. The amount of the shift depends on the height of the sill and on the time elapsed since placement of fill. The profile behind the sill will lower as the fill is eventually carried out of the area behind the sill. As this occurs, the profile will approach its preproject shape.

(2) Aerial photographs.

(a) Aerial photographs can provide quantitative information on shoreline location and a visual qualitative record on the location of underwater shoals, etc. Photogrammetric analysis can provide data on the elevation of the subaerial beach. Aerial photographs may be more readily available for a site than beach profile surveys since it is relatively simple and inexpensive to periodically photograph long stretches of coastline. Many states and Districts routinely obtain such photographs to provide historical records of shoreline changes.

(b) Shoreline location on an aerial photograph depends on the stage of the tide or water level (Great Lakes) and on the level of wave runup at the time the photograph was taken. Wave runup in turn depends on the height and period of the waves and on the beach slope. It is difficult to associate the water level visible on an aerial photograph with a particular datum. The photography could have been taken at low, mean, or high water level, or at any stage in between. Unless tied in stereoscopically with a vertical control datum, the datum will be approximate, especially for historical photographs where information on tidal stage at the time the picture was taken is not available. In addition, photographic distortions may be present that result in variations in scale from one portion of the photograph to another. Rectification of the photography will help to eliminate these distortions. If several sets of aerial photographs spanning several years are available, trends in the shoreline location can be determined. It is often easier to discern the bermline or a debris line associated with high water on an aerial photograph instead of the waterline. The berm line or debris line will give more consistent information regarding beach erosion than will the shoreline. Figure 2-18 shows the

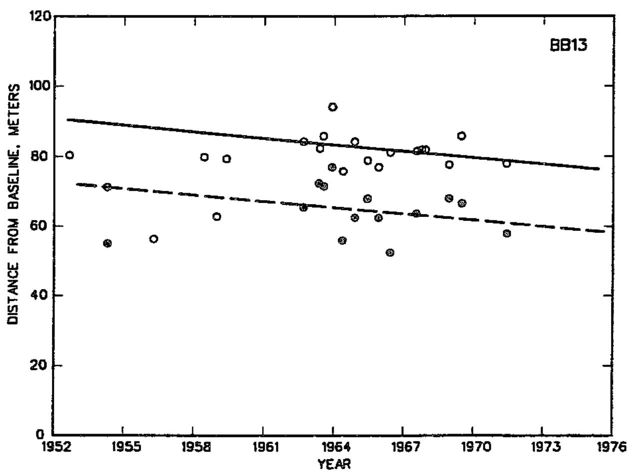


Figure 2-18. Bermline and high-water shoreline location as a function of time, data obtained from aerial photograph analysis (Bradley Beach, New Jersey)

bermline movement on a beach over a 20-year period. The bermline distance was measured relative to an arbitrary baseline located far enough landward so that it is not lost to beach erosion. The line on Figure 2-18 has been fit to the data and suggests slow but steady bermline recession and corresponding beach erosion. (The general beach profile shape has been assumed constant over the 20-year period of analysis.) The scatter of the data points about the trend line is a measure of both the seasonal fluctuations of the bermline (and shoreline) about the long-term trend and the errors involved in determining the bermline location on the photographs.

(3) Other documentation. Other data relating to beach changes include documentation of beach nourishment and sand mining. These might be in the form of tabulated data on volumes of sand placed on, or removed from, a beach or offshore area. For example, operations and maintenance dredging records used for contract payment might provide information on the quantity and location where sand was placed on a beach. Information on the exact distribution of sand along a beach might not be available; however, the quantity and general extent of its placement may be known and may explain observed beach changes found from aerial photograph or beach profile analyses.

f. Sediment budget.

(1) A sediment budget is a quantitative balance of the influx and efflux of sediment within a stretch of beach or other coastal area and the volumetric changes occurring on that stretch of beach. It expresses the conservation of sediment for a coastal cell with specific boundaries stating that the difference in the amount of sediment entering a coastal cell and the amount leaving will cause either erosion or accretion within the cell. If influx exceeds efflux, accretion occurs; if efflux excess influx, erosion occurs. An equation expressing this sediment balance is,

$$Q_{in} - Q_{out} = \frac{\Delta V}{\Delta t} \qquad (2\text{-}13)$$

where

Q_{in} = rate at which sediment is transported into the coastal cell from various sources

Q_{out} = rate at which sediment is transported out of the cell

ΔV = change in sediment volume within the cell

Δt = time period for which the sediment balance is being made

(2) There are generally several sediment sources and several sinks in any sediment budget analysis; Q_{in} and Q_{out} are each composed of several components. Sources of sediment may include longshore transport, cross-shore transport, wind-blown transport, bluff recession, rivers, and man-caused contributions of sand such as beach nourishment. Losses may be by longshore transport, offshore transport, wind-blown transport, transport down offshore canyons, transport into and trapping by tidal inlets, and man-caused losses due to dredging, sand mining, etc. In developing a sediment budget, most of these sources and sinks must be quantified, and the sediment balance equation solved for one unknown. A sediment budget may also balance sediment gains and losses between adjacent beach cells where sand lost from one cell becomes a sand gain for an adjacent cell. In this case, a system of simultaneous equations results (one equation for each cell) that can be solved for the several unknowns. Various assumptions may be made in formulating the equations and choosing what is assumed to be unknown. Typically, a sediment budget is developed for preproject conditions and calibrated using additional data if available. The effects of project construction may be tested by making various assumptions regarding the project's effect on longshore transport, offshore transport, etc. Often, sufficient data may not be available, or the data may not be sufficiently accurate to construct a

sediment budget. For example, small vertical errors in measuring offshore beach profiles can result in large errors in estimating sediment volumes. (A small error in elevation spread over large offshore areas results in large errors in volume.) In such cases, corrections to some components of the sediment budget may be necessary. Results from sediment budget analyses must always be carefully interpreted, and, whenever possible, the sensitivity of results to various assumptions should be tested. A detailed description of sediment budget analyses and their component elements is given in the SPM (1984, Chapter 4, Section VII), and example sediment budgets are given in Weggel and Clark (1983) and EM 1110-2-1502.

g. Other data requirements. Additional data needed for the design of a beach erosion control/stabilization project may include an inventory of existing structures, including their condition and effectiveness; geotechnical data; geophysical data; environmental and ecological data; and historical and/or archeological data.

(1) Existing structures. Data on existing structures might include an inventory of nearby structures and an analysis of their functional performance. The best indication of how a proposed structure will perform is the performance of a similar structure in a similar physical environment. An evaluation of how nearby groins, breakwaters, and sills are performing will provide an indication of how any proposed structures will perform. Also, if there are existing structures within a project area, a decision will have to be made to incorporate them into the project, simply abandon them, or demolish and remove them. This decision will require data on the structural condition and remaining useful life of the structures as well as data on their functional performance.

(2) Geotechnical data. Geotechnical data including the physical properties of underlying soils and their ability to support any proposed structures are required. Many coastal structures such as rubble-mound breakwaters and groins are gravity structures that significantly increase the overburden on underlying soils. Often beach sands are underlain with highly organic, compressible soils that originated in the lagoons behind barrier islands. As the barrier islands migrate landward, the lagoonal sediments appear on the seaward side of the islands. These strata consolidate under load and allow structures founded on them to settle. They may also fail in shear if the project requires that the overburden of sand be excavated to place the structure's foundation. Soil borings are necessary to locate any underlying strata and to obtain samples for testing. Similarly, pile-supported structures such as sheet-pile groins, etc., require data on underlying soil conditions for their design. EM 1110-2-1903, "Bearing Capacity of Soils," and EM 1110-2-2906, "Design of Pile Structures and Foundations," should be consulted for design guidance. In addition, Eckert and Callender (1987) address the geotechnical aspects of coastal structure design.

(3) Geophysical data. Geophysical data such as seismic reflection data can be used in conjunction with offshore core borings to locate and quantify offshore sand resources for beach nourishment. Relatively coarse, good quality sand obtained from offshore sources may provide a more economical alternative than nearshore sources for some beach restoration/stabilization projects.

(4) Environmental and ecological data. In addition to data on physical conditions at a site, baseline environmental data (preconstruction) and environmental monitoring (postconstruction) may be necessary, particularly if the project is expected to adversely impact the environment. Environmental data may include a baseline study of flora and fauna to identify potential environmental impacts that must be considered during the project's design. These baseline data can form a benchmark against which the results of a monitoring study can later be compared to assess the project's impact. A baseline study will identify the flora and fauna indigenous to the project area, identify and locate any endangered species, and provide data that can be used to identify any potentially adverse environmental impacts. Both subaqueous and subaerial communities and the anticipated effect of the beach erosion control/stabilization project on them need to be included. Environmental impacts may also occur at locations remote from the actual project, for example, at the sources of beach fill and construction materials. Beach-fill sand obtained from offshore requires dredging and thus affects bottom dwelling organisms. Environmental studies must be tailored to the specific needs of a given project. Additional guidance can be found in EM 1110-2-1204.

(5) Historical and archeological data. An archeological investigation might also be indicated if the proposed project is suspected to be near a historical site. This investigation would identify, map, and restrict access to historical or archeological areas endangered by the project.

2-3. Detached Breakwater and Groin Databases

a. Breakwater database. The US Army Engineer Waterways Experiment Station's CERC maintains a database of detached breakwater projects in the United States and several other countries. The database includes information on the type and purpose of the breakwater, its date of construction, and various project dimensions. Information on the physical environment at the site is also provided along with a brief narrative description of the project's performance and any unique features.

b. Groin database. A similar database is being developed for groins; however, because of the large number of groin projects in the United States, a complete listing is not available. Only those projects having some unique feature are included.